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ELASTIC-PLASTIC SEISMIC ANALYSIS OF POWER PLANT BRACED FRAMES

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T.A. Nelson† and R.C. Murray†

ABSTRACT

The Lawrence Livermore Laboratory was asked by the U.S. Nuclear Regulatory Commission (NRC) to investigate the inelastic behavior of a representative noncategory I structure to determine the amount of reserve seismic capacity that is available beyond elastic design levels.

Elastic and elastic-plastic seismic analyses were conducted on a braced steel frame using eight time history records. In addition, two spectra were used in elastic analyses only. By comparing elastic limit response with the ultimate capacity, the reserve strength of the frame was determined. To ensure operability, a frame model incorporating a piping system was subjected to the above seismic loadings using elastic analyses.

It was found that the piping system components controlled the seismic capacity of the combined structure. The mean reserve capacity was 2.6 times the seismic design level based on the time history analyses.

Keywords: Braced Frame, Dynamics, Earthquakes, Frames, Inelastic Behavior, Piping, Power Plants, Seismic Analysis, Structural Engineering

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INTRODUCTION AND SUMMARY

Accompanying the advancement in scientific knowledge of earthquake phenomena are changes in seismic design and performance criteria for structures. Thus, the earthquake induced force levels predicted by present techniques may be significantly higher than those for which power plant structures were designed and built. However, it is generally recognized that there is considerable conservatism present in commonly employed elastic design methods. When structural elements are allowed to respond in the inelastic range they cause seismic energy dissipation and force redistribution in a structure. This will often reveal a reserve capacity in the structure beyond that predicted by an elastic analysis. A knowledge of the inelastic behavior of typical structural configurations used in operating plants can assist in determining their actual safe seismic capacity.

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A typical diagonal braced steel frame was developed to determine the amount of reserve capacity that is available beyond elastic design levels. The frame was analyzed first using elastic static and dynamic analyses. The loadings included dead and live load, an equivalent static lateral earthquake load, two response spectra and a suite of eight earthquake time history records. The response spectra used were the Housner and Regulatory Guide 1.60. The time histories represented different site conditions, distances to causative faults and magnitudes. The lateral static load and Housner spectrum represent vintage design criteria, while the R.G. 1.60 and time history analyses reflect current methodology. The elastic limit responses of the structure were determined along with the accompanying threshold peak ground accelerations (threshold g values).

The frame was then analyzed using two-dimensional elastic-plastic analyses for the eight time histories. From these analyses, the peak ground accelerations corresponding to the ultimate capacity of the frame were determined (ultimate g values). By comparing the threshold g values with the ultimate g values, the reserve capacity of the structure was determined.

The analysis of the frame represents only one aspect of the problem. To remain functional, the piping and equipment contained within the structure must be operable during and after a seismic event. Therefore, a representative piping system was developed that consisted of vertical and horizontal runs of 8- and 10-in. piping, a pump, valves, elbows, tees, and a reducer. A model incorporating the frame and piping system was subjected to the eight time histories and two spectra using elastic analyses. The threshold g values for operability of each component were calculated based on the limits specified by manufacturers data or the ASME Boiler and Pressure Vessel Code.[1]

The results of the original study[2] conducted for the Nuclear Regulatory Commission will be used to generate guidelines for using inelastic response criteria to assess more accurately the actual safe seismic load capacity of operating reactor components and structures. It is not intended that the reserve capacity of this frame or piping system will apply directly to others of the same type. Rather, the results and conclusions presented should serve as a basis for accounting for what appears to be a significant amount of reserve capacity in structural systems.

The results of the study show that the braced frame alone has a reserve capacity more than five times greater than the design level. The design level response was controlled by buckling of the bracing, and the ultimate level response was controlled by a 1-foot deflection limitation. Both limiting levels were based on the mean response to the eight time histories. The analysis of the combined frame and piping indicated that the pump was the controlling element of the frame and pipe system, reducing the total available reserve capacity to 2.6 times the design level. The R.G. 1.60 spectra were the most conservative input for a specified peak ground acceleration, as was to be expected since their amplification and broad frequency band are based on response at a mean plus one standard deviation level. The Housner spectrum provided the closest results to the mean response of the eight records. The static lateral load, distributed as specified in the Uniform Building Code (UBC),[3] was the least conservative as compared to the mean.

SELECTION OF LOADING

To conduct elastic-plastic seismic analyses of a structure, time histories of support motion are required for input. The series of earthquake loadings applied to the frame and frame and pipe system are illustrated in Fig. 1. Because the dynamic response of structures is sensitive to the frequency content, phasing, magnitude, and duration of the input, it was felt that a suite of actual recorded time histories would be required. For this study, a vertical and a horizontal component from each of eight earthquake records was chosen; these are listed in Table 1 and include records of different magnitude, epicentral distance, peak acceleration, and site conditions.

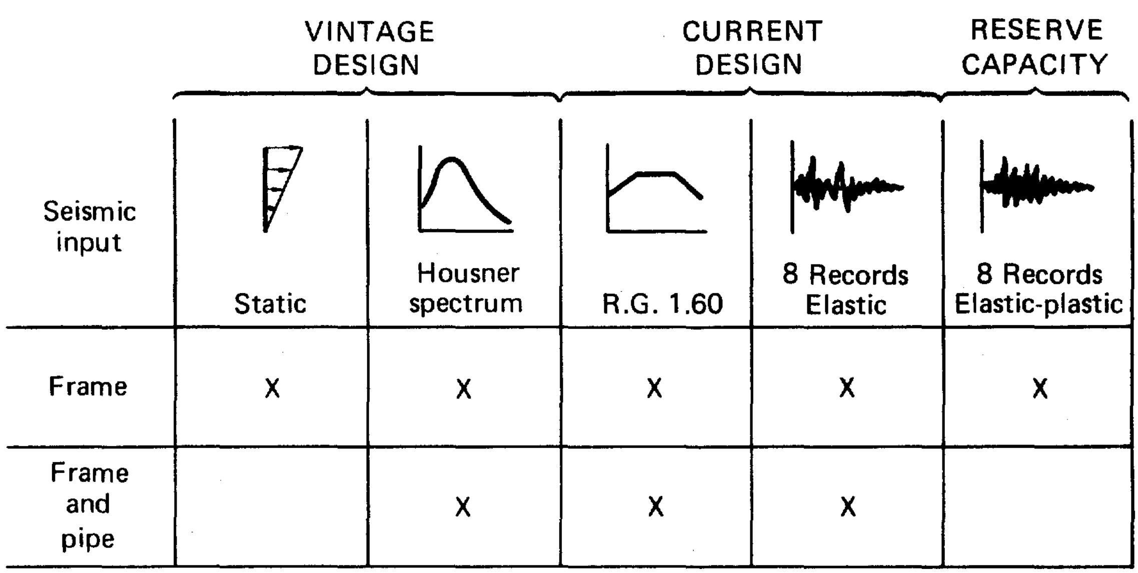

FIG. 1. Summary of earthquake input.

TABLE 1. Selected earthquake records.

Type	Code	Name	Peak Acceleration (g)	V/H Ratio	Magnitude	Epicentral Distance (km)	Site
Far field	F1	Kern Co. 1952 Santa Barbara Court House	S48E 0.129 V 0.044	0.34	7.6	90	
	F2	El Centro Site 2/9/56	S90W 0.05 V 0.012	0.24	6.8	126	Deep soil
	F3	Seattle 4/13/49 Dist. Eng. Office	N88W 0.066 V 0.022	0.33	7.1	60	Soil
Inter-mediate distance	I1	Kern Co. 1952 Taft Lincoln School Tunnel	S69E 0.176 V 0.103	0.58	7.6	43	Stiff soil
	I2	Eureka 12/21/54 Ferndale City Hall	N46W 0.20 V 0.043	0.22	6.5	40	Stiff soil
Near field	N1	El Centro 5/18/40 Station 117	S00E 0.34 V 0.20	0.59	6.7	10	Deep soil
	N2	Helena, Montana, 10/31/35 Carroll College	S90W 0.146 V 0.089	0.61	6.0	10	Rock
	N3	San Fernando 2/9/71 Pacoima Dam	S16E 1.15 V 0.69	0.60	6.6	10	Rock

For analyzing the frame, static vertical loadings were defined that represent the dead and live loading. The floor and roof dead load consisted of the weights of the girders plus a 7.5-in. concrete slab. The live loading included a 30-psf snow load on the roof and a 100-psf equipment load on the floors. These loads were included in the total mass of the structure for seismic analysis.

DEVELOPMENT OF STRUCTURAL MODELS

Because the objective of this study was to address generic issues of analysis, the ideal model would have been one that would represent all types of structures encountered in an operating power plant. Because this was not possible, a steel frame, as might be used in a noncategory I structure, was chosen based on a 2-story, 3-bay diagonal braced frame from an operating plant. The basic frame is shown in Fig. 2. This represents an interior frame having a bay spacing of 29.3 ft. The connections at the ends of the bracing and beam members are considered to be pinned.

The frame was idealized as an assembly of 20 beam-column elements and 4 truss elements. The mass contributions from dead and live load were lumped at the 18 nodes shown in Fig. 2. This model was used for elastic and elastic-plastic analyses. SAP4[4] was used for the elastic analyses using straightforward elastic element formulations. The program DRAIN-2D[5] was used to analyze the structure for inelastic behavior in which the material characteristics were modeled by specifying an elastic-plastic stress-strain relation.

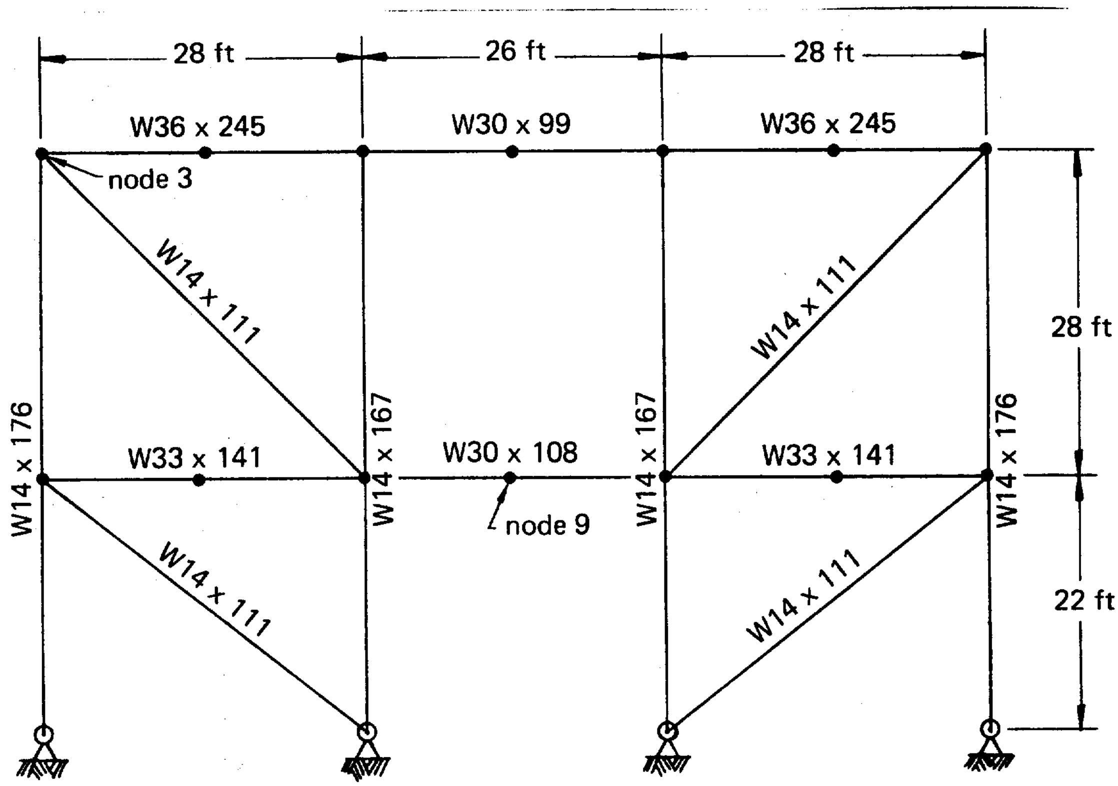

FIG. 2. Model of braced steel frame.

The objective of the piping system model was to investigate the response of a composite frame and pipe system. This was to allow a determination of the effect on the frame of vertical and horizontal runs of different diameter piping, and to show the limitations imposed by the operability requirements of associated equipment such as pumps and valves. The mathematical model of the pipe layout is shown in Fig. 3.

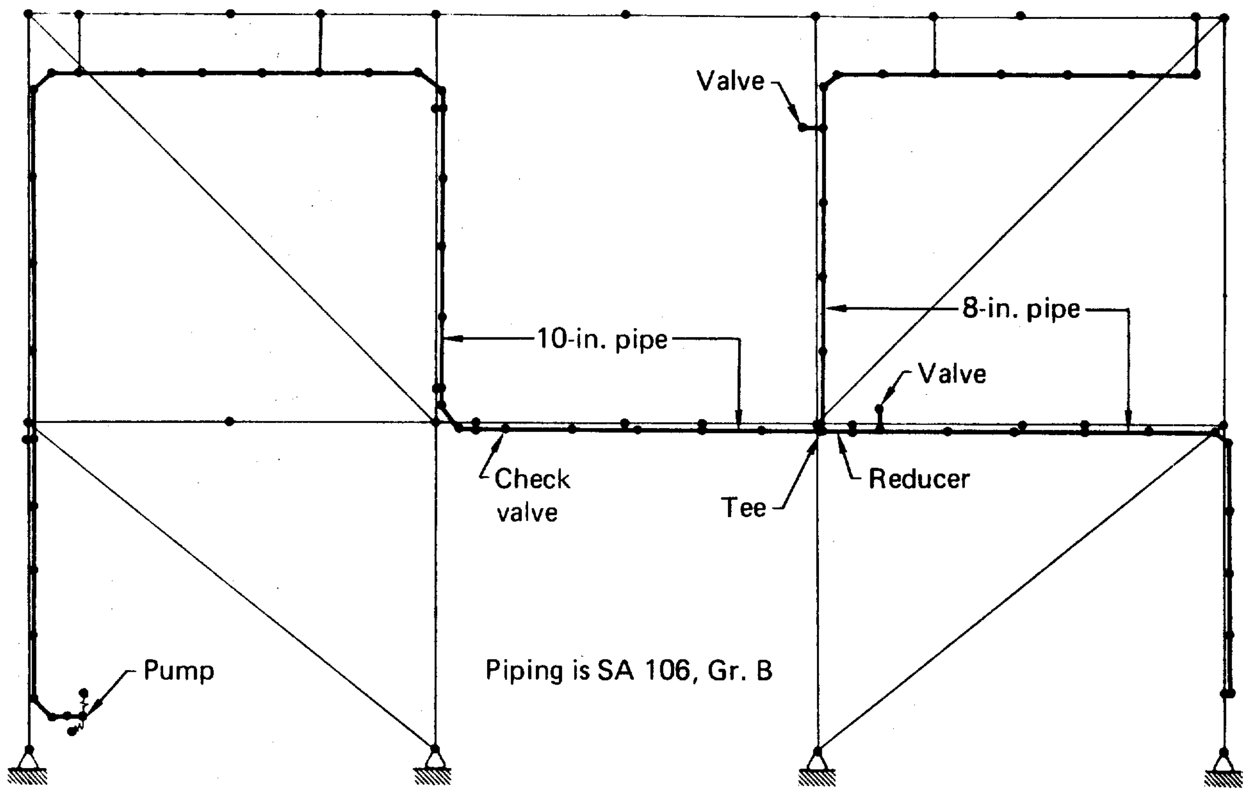

FIG. 3 Mathematical model of frame and pipe system.

The contributing masses were lumped at the nodes as shown. The weight of the contained water and the pipe insulation was distributed to the nodes on a tributary area basis. The piping was assumed to have a line pressure of 600 psi and a temperature of 400°F. The valves were modeled as nodes on the pipe with their associated lumped mass. The valve actuators were modeled as eccentric masses attached to the main pipe run by pipe elements. The pump was modeled with SAP4 boundary elements representing two linear stiffnesses and one rotational stiffness.

STRUCTURAL ANALYSIS

The analyses of the frame and the frame and pipe system entailed several steps. The frame model alone was subjected to the following:

- Apply fixed-end force set including the effects of dead and live loads
- Analyze for static vertical and lateral loads
- Conduct linearly elastic dynamic analyses using two spectra and eight time histories
- Combine dead and live load results with seismic results, and determine threshold accelerations for elastic behavior
- Conduct inelastic time history analyses using several different scaling factors times each of the eight time histories with the dead and live load superimposed
- Determine the peak ground acceleration values associated with the ultimate level behavior of the frame
- Compare elastic and elastic-plastic results, and calculate reserve capacity due to inelastic response

The frame and pipe model was subjected to a similar set of analyses. However, because the performance criteria for the pipe differed from those for the frame, only elastic analyses were conducted for the combined frame and pipe system. In addition to the operating loads applied to the frame, the pipe was subjected to thermal and pressure loadings.

LIMITING CRITERIA

Limiting criteria were established to measure structural performance at different levels of response. The following levels were considered:

- Elastic design level
- Limit of elastic response
- Ultimate capacity
- Equipment service limits corresponding to upset or emergency operating conditions.

For the elastic analyses of the frame, the performance of the bracing members in compression was the controlling factor. The elastic design level performance defined by the American Institute of Steel Construction (AISC)[6] specified buckling levels with the appropriate safety factors. The 33% overstress allowance for earthquake loading was not included. Note that the limit of elastic response is not reached until the member is subjected to the unfactored buckling stress. Thus, the safety factor provides reserve capacity even within the elastic response range.

After reaching the limit of elastic response, the structural members respond in either a buckling or yield mode. The model employed is based on the assumption that the bracing members yield in tension and buckle before yielding in compression, i.e., they exhibit incomplete hysteretic action.

For the elastic-plastic analyses, the limiting criteria are composed of two parts. It was decided that the serviceability of the structure would be limited by excessive deflections. A 1-foot lateral deflection at the roof level of the frame was chosen as a limiting value. The other criterion to consider was the strain capacity of the material. As it turned out, a 1-foot lateral deflection at node 3 (Fig. 2) results in a maximum extension in a bracing member corresponding to a 2% strain. This is slightly beyond the strain hardening point, and, thus, was considered to be a reasonable limiting value. During the analyses, it was discovered that the frame exhibited unusual behavior for certain earthquake loadings. The vertical component was causing a plastic hinge to form at node 9 in the center of a floor member, which affected the lateral behavior of the frame. This anomalous behavior seemed to occur when the vertical deflection at node 9 exceeded 1-foot. Therefore, the limiting criteria were adjusted to include a 1-foot limit on vertical deflection as well.

The emphasis for the frame and pipe system was on the operability of the piping system components. The ASME code, Section III, provided the primary criteria for the piping itself. The piping was assumed to be Class 2. It was to survive the earthquake loading at Level C service limit stresses.

The operability limits for the valves and pump are more difficult to quantify for a generic study. Usually these components would be chosen and certified on the basis of an expected load level for a specific plant. However, it was decided that reasonably representative values could be devised. Commonly, operability limits for motor operated valves are specified on the basis of peak inertial loading of the valve body. Two common levels of limiting acceleration are 3 g and 5 g. Thus, threshold ground accelerations were calculated for both of these limiting levels. For the pump, it was decided that the nozzle forces should be limited to a stress that could be resisted without damage requiring repair. This stress level corresponds to a Level B service limit as specified in the ASME code. The philosophy employed was that the pump should be able to resist nozzle forces of at least Level B stresses in the pipe attached to it.

RESULTS

Braced Frame Elastic Analyses

The resulting threshold g values for the elastic level responses of the braced frame are shown in Table 2. The factored buckling level includes the safety factor specified in AISC specifications and indicates a design limit. The unfactored buckling level indicates the theoretical buckling level and, thus, incipient inelastic response. Note that there is a rather wide variation in response to the different input records, some of which required large multiplication factors.

TABLE 2. Threshold accelerations for buckling of bracing members.

Record	Peak Ground Acceleration as Input (g)	Maximum Bracing Stress[a] (due to EQ) (ksf)	AISC-Factored Buckling Threshold g (g)	AISC-Factored Buckling Mean Threshold g (g)	Unfactored Buckling Threshold	Unfactored Buckling Mean Threshold g (g)
F1	0.129	416	0.46		0.90	
F2	0.05	188	0.40	0.40	0.79	0.79
F3	0.066	293	0.34		0.67	
I1	0.176	763	0.35	0.51	0.68	1.00
I2	0.20	450	0.67		1.31	
N1	0.34	1492	0.34		0.67	
N2	0.146	814	0.27	0.36	0.53	0.71
N3	1.15	3651	0.47		0.93	
All Time Histories				0.41		0.81
Housner	0.124	432	0.43		0.85	
R. G. 160	1.0	6662	0.23		0.44	
UBC	1.0	2413	0.62		1.23	

[a]In first story bracing members

The mean threshold g for all the records turned out to be nearly the same as for the far-field records, approximately 0.4 g. This was at about the same level as predicted by the Housner spectrum. The R.G. 1.60 spectra showed the most conservative threshold acceleration at 0.23 g, but is based on a mean plus one standard deviation level. The corresponding level (i.e., mean $+1\sigma$) for the accelerogram records was 0.29 g, which is at least in the same vicinity. Note, however, that the R.G. 1.60 spectra are based on a larger sample.

Braced Frame Inelastic Analyses

The next level of reserve capacity to be evaluated was that due to ineleastic response of the frame. Again, the first story bracing members played a major role in the structure's response. Because these members provided the primary lateral restraint to the frame, their yielding provided the major energy absorption mechanism.

As stated earlier, the frame was subjected to several different peak ground acceleration levels for each earthquake record. A 1-foot horizontal deflection at node 3 or a 1-ft vertical deflection at node 9 indicated that the capacity of the frame had been reached.

A summary of the ultimate g values for these two failure criteria are shown in Table 3. The average minimum reserve capacity in the frame alone is a factor of 5.4 over design values. This total reserve capacity is composed of two parts: the elastic safety factor against buckling (1.914); and a factor due to ductile action, which averages 2.8 times. This corresponds to a local ductility of 16 in the lower bracing members.

TABLE 3. Ultimate peak ground accelerations and reserve capacity for the frame.

Record	Design[a] Threshold g (g)	Horizontal (Node 3) Value Controls		Vertical (Node 9) Value Controls		Minimum Reserve Capacity[b]
		Ultimate g (g)	Reserve Capacity[b]	Ultimate g (g)	Reserve Capacity[b]	
F1	0.46	2.4	5.2	1.9	4.1	4.1
F2	0.40	1.7	4.2	1.8	4.5	4.2
F3	0.34	1.9	5.6	1.6	4.7	4.7
I1	0.35	3.0	8.6	1.3	3.7	3.7
I2	0.67	2.6	3.9	2.6	3.9	3.9
N1	0.34	3.3	9.7	2.5	7.4	7.4
N2	0.27	---	---	2.8	10.4	10.4
N3	0.47	3.7	7.9	2.2	4.7	4.7
Mean	0.41	2.7	6.4	2.1	5.4	5.4
(σ)	(0.12)	(0.73)	(2.3)	(0.52)	(2.3)	(2.3)

[a]AISC factored buckling, which would be used for original design.
[b]Reserve capacity = (ultimate g)/(design threshold g).

Frame and Pipe System

The threshold peak ground accelerations on reaching designated limiting criteria are delineated in Table 4, and the mean values are shown in Table 5. The same trends are exhibited for the frame and pipe system as for the frame alone. The standard deviation of the threshold accelerations becomes quite large for the piping components, usually resulting from the high threshold g levels in response to record I2. Because the piping response to this record is significantly different from the other time history responses, it was decided to omit record I2 and use the mean of the other seven time histories. This is considered to be the most reasonable option as these low responses probably result from particular idiosyncrasies of frame and piping system. For generic results these idiosyncrasies should be bypassed insofar as possible.

The results show that the piping system causes about a 20% lower threshold g for bracing member buckling than was the case with the frame alone, resulting primarily from an increased inertial load imparted by the mass of the piping system. Because the assumed live load applied to the frame alone was sufficiently large to include a generic piping system, the results from that analysis were used for the frame response. The frame and pipe system analysis will be used for the evaluation of piping components only.

TABLE 4. Threshold peak ground accelerations (g) for the frame and pipe system components.

Component (Record)	Pump[a]	Valve		Tee[c]	Straight Pipe[c]		Reducer[c]	Elbow[c]	
		3g[b]	5g[b]	8×10×10 in.	10 in.	8 in.	10;8 in.	8 in.	10 in.
F1	1.48	1.48	2.47	1.83	2.35	2.04	2.34	3.28	3.70
F2	1.10	1.12	1.88	1.30	1.82	1.81	2.08	3.02	3.03
F3	0.78	0.92	1.54	1.12	1.30	2.03	2.32	2.29	3.12
I1	0.87	1.00	1.68	1.02	1.44	1.35	1.55	2.00	2.52
I2	1.74	1.87	3.12	2.36	2.87	5.97	6.85	5.64	5.06
N1	0.82	0.93	1.55	1.15	1.36	2.18	2.50	2.37	2.28
N2	0.68	0.78	1.31	1.03	1.13	1.66	1.90	1.80	1.73
N3	1.02	0.94	1.57	1.26	1.68	1.58	1.82	2.42	2.65
Housner spectrum	1.06	0.82	1.35	1.02	1.75	2.06	2.36	2.07	2.69
R.G. 1.60 spectra	0.56	0.45	0.75	0.56	0.93	1.10	1.26	1.13	1.48

[a]Level B stress limit.

[b]Limiting acceleration levels.

[c]Level C stress limit.

TABLE 5. Mean values of threshold acceleration (g) for the time history inputs.

Component	Pump	Valve		Tee	Straight Pipe		Reducer	Elbow	
		3g[a]	5g[a]	8×10×10 in.	10 in.	8 in.	10×8 in.	3 in.	10 in.
Far field	1.12	1.17	1.96	1.42	1.82	1.96	2.25	2.86	3.28
Intermediate	1.31	1.44	2.40	1.69	2.16	3.66	4.20	3.82	3.79
Near field	0.84	0.88	1.48	1.15	1.39	1.81	2.07	2.20	2.22
All T-H records	1.06	1.13	1.89	1.38	1.74	2.33	2.67	2.85	3.01
(σ)	(0.37)	(0.36)	(0.61	(0.47)	(0.59)	(1.50)	(1.72)	(1.23)	(1.02)
Without record I2	0.96	1.02	1.71	1.24	1.58	1.81	2.07	2.45	2.72
(σ)	(0.27)	(0.23)	(0.37)	(0.28)	(0.41)	(0.30)	(0.34)	(0.53)	(0.64)

[a]Limiting acceleration levels.

CONCLUSIONS

The results show that a substantial reserve capacity is available in both the structure and the attached piping. The total capacity of the system is governed by the pump. The mean reserve capacities available in each portion over the design level of the frame are shown in Table 6. The design level of the frame was taken to be the threshold g at the factored buckling level for each earthquake record (Table 2). The reserve capacities were calculated for each record, then averaged. The mean values shown do not include record I2 in the calculation. However, inclusion of I2 would not affect the reserve capacity numbers until the 8-in. straight pipe controlled.

TABLE 6. Reserve capacities.

Item	Limiting Criterion	Mean Ratio[a]
Pump	Level B stress limit at nozzle	2.6
Valves	3-g acceleration	2.7
Tee	Level C stress	3.3
Pipe	Level C stress	4.2 - 5.0
Frame	1-ft deflection	5.4
Reducer & elbows	Level C stress	5.7 - 7.2

[a]Mean ratio of the capacity resulting from an item to the code design capacity of the frame.

Thus, based on operability considerations, the frame and pipe system is shown to have a reserve capacity at least 2.6 times its design level using recorded earthquakes as the design basis. If the frame system were designed using the Housner spectrum, the mean time history results would suggest a reserve capacity of 2.2 times design. If the R.G. 1.60 spectra were the design basis, these analyses would predict a reserve capacity of 4.2.

For operability considerations, only elastic analyses were employed. The overall response of the frame was still linear in the acceleration range wherethe pump and valves reach their operability limit. This suggests that elastic analyses of the frame and pipe system are adequate to establish operability limits in this case, and that very little would be gained by looking at the inelastic response of the combined system. This may not be true in general, however.

It should be noted that conclusions of a generic nature based on the analysis of one frame and pipe system subjected to a limited number of earthquake records are somewhat tenuous. However, the results presented herein show that it is quite reasonable to assume that significant reserve capacity is available beyond that predicted by elastic analysis. The original study[2] should provide the basis for using reserve capacity in structures and indicates the potential effect of attached equipment. Some of the considerations essential to the use of the reserve capacity of nuclear power plant structures are presented in Ref. 7.

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